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VARIATION OF SEISMIC RESPONSE OF MID-RISE RC BUILDINGS DUE TO SOIL STRUCTURE INTERACTION EFFECTS

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ABSTRACT

The seismic design of RC buildings requires determining the expected base shear, lateral drift at each story level and internal forces of the structural elements. In the analysis, it is common for the structural engineers to consider a fixed base structure which means that the foundations and the underlying soil are assumed to be infinitely rigid. This assumption is not proper since the underlying soil in the near field often consists of soft soil layers that possess different properties and may behave nonlinearly leading to drastic variation of the seismic motion before hitting the structure foundation. In addition, the mutual interaction between the structure, its foundation and the underlying soil during the vibrations can substantially alter the structure response. This response variation depends on the structure characteristics, the soil properties and the nature of the seismic excitation. Consequently, an accurate assessment of inertial forces and displacements in structures requires a rational treatment of soil structure interaction (SSI) effects. In this paper, comprehensive numerical study is carried out to investigate the seismic response of mid-rise RC buildings subjected to different seismic excitations assuming full nonlinear SSI employing PLAXIS V8.2 software. Three types of two dimensional mid-rise moment resisting frames consisting of five story (S5), ten story (S10) and fifteen story (S15) are analyzed. Each building is considered to be founded on

three types of soil representing firm soil (class A), medium soil (class C), and loose soil (class D) conditions with shear wave velocity (Vs) of 1000, 270, and 90 m/s, respectively. For comparison, each building intermediate frame has been analyzed with different base boundary conditions assuming: (i) fixed base; (ii) equivalent soil springs; (iii) flexible base considering full SSI. The results showed that it is essential to consider SSI effects in the procedures of the seismic design of concrete mid-rise moment-resisting frames. Generally, decreasing the dynamic stiffness of the subsoil (by decreasing Vs and shear modulus G) the base shear ratios decrease while inter-story drifts of the frames increase relatively. Moreover, assuming fixed base can lead to high overestimation of the structure design forces and seismic response.

Key words: Mid-Rise RC Buildings, Soil Structure Interaction (SSI), Seismic Response, Nonlinear Dynamic Analysis.

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1. INTRODUCTION

The soil structure interaction (SSI) refers to the action in which the response of the soil influences, the response of the structure and the response of the structure influences the motion of the soil. The importance of the SSI for static and dynamic problems has been very well established and the literature covers at least 30 years of computational and analytical methods to solving SSI problems. Considering dynamic SSI effects enables the designer to estimate the inertial forces and real displacements of the soil-foundation structure system precisely under the influence of free-field motion. For flexible or small buildings supported on firm soil the effects of the interactions are usually insignificant, while the interactions of stiff and heavy structures located on soft soil are very critical. Since the 1990s, great effort has been made to substitute the classical design methods by the new ones based on the concept of performance-based seismic design. Moreover, structural damages during the earthquake in Mexico City in 1985 and many other recent earthquakes, such as those in Christchurch in 2011 (New Zealand), Japan in 2011 (Fukushima) and Nepal earthquake in 2015, clearly demonstrate the crucial effects of local soil properties on the earthquake response of structures. Therefore, there is a strong engineering motivation for a site-dependent dynamic response analysis to determine the free-field earthquake motions. The determination of a realistic site-dependent free-field surface motion at the base of the structure can be the most important step in the earthquake-resistant design of structures.

When SSI is taken into consideration, the ground motions imposed at the foundation of the structure are influenced by the soil properties, travel path, local site effects, and the geometry of the soil medium. Wolf and Deeks, 2004 [1] summarized the four basic SSI effects on structural response as: (i) increase in the natural period of the system, (ii) increase in the damping of the system, (iii) increase in the lateral displacements of the structure, and (iv) change in the base shear depending on the frequency content of the input motion and dynamic characteristics of the soil and the structure. Veletsos and Meek, 1974 [2] concluded that SSI has two basic effects on structural response compared with the fixed base counterpart: (1) the soil-structure

system has an increased number of degrees of freedom and thus modified dynamic characteristics; (2) A significant part of the vibration energy of the soil-structure system may be dissipated either by radiating waves, or by hysteretic material damping in the soil. Several researchers [3-6] have studied the structural behavior of unbraced structures subjected to earthquake under the influence of the SSI. Examples are given in Gazetas and Mylonakis, 1998 [3] including evidence that some structures founded on soft soils are vulnerable to SSI. Dutta et al., 2004 [7] found that for low-rise unbraced buildings, the lateral natural period is very small and may lie within the sharply increasing zone of response spectrum. Hence, an increase in lateral natural period due to the effect of soil-structure interaction may cause an increase in the spectral acceleration ordinate. Therefore, they concluded that the effect of soil-structure interaction may play a significant role in increasing the seismic base shear of low-rise building frames. However, seismic response generally decreases due to the influence of SSI for medium to high rise buildings. Galal and Naimi, 2008 [4] mentioned that the effects of SSI on the seismic performance of concrete moment resisting building frames up to 20 stories, resting on soft and medium soil types, are significant while those effects are negligible for stiff soils and rocks. Recent studies [8] proved that while considering soil in the analysis of building frame, the full 100% fixity may not be ensured because the settlement and rotation of foundation considerably alter the behavior of building frame.

In recent years, several efforts have been made for developing analytical methods to assess the structure responses and supporting soil media under seismic excitations. Successful application of these methods is vitally dependent on the incorporation of the soil properties in the analyses. Therefore, substantial effort has also been made toward the determination of soil attributes to use in these procedures [9]. The main two analytical procedures for dynamic analysis of soil-structure systems under seismic loads are the equivalent-linear method and the fully nonlinear method. Byrne and Wijewickreme, 2006 [10] provided an overviews of the mentioned methods and discussed the benefit of the fully nonlinear method over the equivalent-linear method in various practical applications. Their research results proved that the equivalent linear method is not appropriate to use in dynamic SSI analysis; it does not directly capture all nonlinearity effect because it assumes linear behavior during the solution process. Moreover, the strain-dependent modulus and damping functions are only taken into account in an average sense that means approximation of some nonlinearity effects. Therefore, they concluded that the most appropriate method for dynamic analysis of a soil-structure system is a fully nonlinear method. This method correctly represents the physical properties and follows any stress-strain relationships in a realistic way. In addition, Lu et al., 2011 [11] illustrated the potential for further reliance on computer simulations in the assessment of the nonlinear seismic ground response using nonlinear dynamic analysis. Based on the aforementioned priorities and capabilities of the fully nonlinear method this method is adopted in this study to attain rigorous and reliable results for dynamic analysis of soil-structure systems.

In this paper, comprehensive numerical study is carried out to investigate the seismic behavior of mid-rise reinforced concrete buildings subjected to different seismic excitations considering full nonlinear SSI employing PLAXIS V8.2 software [12, 13]. Three types of two dimensional moment resisting RC frames with different heights to represent the traditional mid-rise buildings are considered for the analysis. The first frame consists of five story (S5), the second of ten story (S10) and the third of fifteen story (S15). Each building is assumed to be founded on three different types of soil throughout the current analysis to represent the firm soil condition (class A),

medium soil condition (class C) and loose soil condition (class D) as described by Egyptian code for calculating the loads and forces, ECP-201, 2012 [14]. Three different seismic records with various frequency contents are employed for the excitations. Moreover, each building intermediate frame has been analyzed assuming different base boundary conditions assuming fixed base and flexible base considering full SSI to compare the results and find out the SSI effects.

2. NUMERICAL SIMULATION OF THE SOIL-STRUCTURE SYSTEM

2.1. Model Description

The considered soil structure system along with the finite element mesh discretization is illustrated in Fig. 1. PLAXIS 2D V8.2 software is utilized for modeling and analysis of the soil-structure system. The soil-structure model was comprised of plate elements to model the beams, columns, and raft foundation of the structure frame elements. The 2D plane strain triangular element is used to model the soil medium and the rigid boundaries to model the bedrock. The interface element is used to simulate frictional contact and probable slip as a result of seismic excitation. The properties of interface elements are assumed to be same as of soil properties. The Mohr-Coulomb model has been adopted in this study as the constitutive model in the soil-structure model to simulate the nonlinear behavior of the soil medium. The Mohr-Coulomb model is an elastic-perfectly plastic model that has been used by many researchers [15,16] in modeling the dynamic SSI to simulate soil behavior under seismic excitations in soil-structure systems. In numerical analysis, it is essential to account for the radiation conditions through efficient techniques to avoid spurious wave reflections at the mesh boundaries [17]. Therefore, for the lateral boundaries of the soil medium, the viscous absorbent boundaries developed by Lysmer and Kuhlemeyer, 1969 [18] were used. The proposed method is based on using independent dashpots in the normal and shear directions at the model boundaries.

The horizontal distance between soil boundaries is assumed to be 250m. The vertical depth of soil is assumed to be 75m. To obtain the desired accuracy with a reasonable computing time requirements, the accuracy of different FE meshes adopted with energy absorbing boundaries of Lysmer type, is verified by trial-and-error method to achieve a considerable reduction in computational domain. In order to transmit all the vibratory wave patterns, the plastic deformations are expected to be formed [19]. This is achieved by employing smaller element size ($\Delta h \leq 1$ m), to verify the condition that the element size should be one-eighth to one-fifth of the shortest Rayleigh wavelength at the highest frequency included in the Fourier response spectrum of the excitation. The time step integration has been chosen as step of input motion (Δt) taking into account the Courant condition for the FEM simulations [20]. The soil domain is divided into three regions; first region with horizontal length L1=60m and vertical depth H1=15m with fine mesh discretization; second with L2=140m and H2=40m with relatively coarser mesh; third with L3=250m, H3=75m with coarse mesh discretization as shown in Fig. 1. Many researchers concluded that the rigid boundary condition is the most appropriate and realistic condition for bedrock modeling in dynamic soil-structure analysis [7,21]. As mentioned in the previous section, three types of underlying soil were considered. The first soil type is firm soil, with shear wave velocity (Vs) of 1000 m/s to represent soil class A as described by the Egyptian code [14]. The second type is medium soil with Vs of 270

m/s to represent soil class C. The third type is loose soil condition with Vs of 90 m/s to represent soil class D. In each case of the analysis, the three underlying soil regions were considered to be uniform and formed from the same class. The soil classes, properties and parameters used for the input data are given in Table 1 [19]. In addition, the boundary condition for bedrock was assumed to be rigid in numerical analyses conducted by other researchers [22-24].

2.2. RC MOMENT RESISTANT FRAME CHARACTERISTICS

Three reinforced concrete intermediate moment resistant frames with different heights to represent the mid-rise residential buildings were designed for the investigation. The first building consists of five story and referenced herein as S5, the second building with ten story is referenced as S10 and the third building has 15 story that referenced as S15.The typical story height is considered to be 3.0 meters and each building has one basement of 2.0 meters height. Each intermediate frame consists of 3 bays of 4.0 meters width to give a total frame width of 12.0 meters. The spacing between frames is assumed to be 5.0 meters as given in table 2. The cross sections of frame columns and floor beams were preliminary designed according to the requirements of the Egyptian code for design and construction of reinforced concrete structures, ECP-203, 2007 [25]. The permanent (dead) load and imposed (live) load are determined as uniform distributed loads over the beams. In this study, the total loads on each beam were considered to be 50 KN/m. The dimensions and characteristics of frames are illustrated in Fig. 2. The foundation is assumed to be a raft foundation with thickness of 0.60m for S5, 1.0m for S10 and 1.5m for S15. Because this is a plane strain problem, the foundation width is taken to be 5.0m equal to the frame spacing to calculate the moment of inertia of the concrete element only. It is assumed that the concrete has a characteristic compressive strength (f_{cu}) of 25Mpa and the modulus of elasticity of concrete (Ec) is calculated as Ec = 4400 $\sqrt{Fcu}$ [25].

Table 1 Soil classes, properties and assumed parameters

Soil types	**Unit weight γ (KN.m^{-3})**	**Shear modulus G (KN.m^{-2})**	**Poisson's ratio (υ)**	**Shear wave velocity Vs (m.s^{-1})**	**Cohesion (c) (KN.m^{-2})**	**Friction angle (φ) (°)**	**Interface strength reduction factor (R_{inter})**
Firm class A)(	20.64	2.10 E+06	0.35	1000.0	30.0	38.0	0.67
Medium (class C)	18.64	1.38 E+05	0.30	270.0	0.0	35.0	0.67
Loose class D)(	16.67	1.38 E+04	0.25	90.0	0.0	33.0	0.67

Table 2 Dimensions of the moment resisting building frames

Building Reference	**Number of stories**	**Number of bays**	**Story Height (m)**	**Bay Width (m)**	**Total Height (m)**	**Total Width (m)**	**Spacing of frames**
S5	5	3	3 for typical + 2 for basement	4	14	12	5
S10	10	3		4	29	12	5
S15	15	3		4	44	12	5

2.3. Input Motion Characteristics

To find out the effect of the seismic excitation characteristics on the response of the soil structure system, three different input motions in the form of recorded accelerogram were selected and employed for the time domain analysis as summarized in table 3 [26]. Naumoski at al., 1988 [27] classified excitations according to their acceleration to velocity ratios (A/V) as; high A/V ratio where A/V>1.2, intermediate A/V ratio with 0.8 < A/V <1.2, and low A/V ratio in which A/V < 0.8, where A is maximum acceleration in g, and V is maximum velocity in m/s. The term A/V gives a direct indication to the frequency content of motion in the same manner. The low frequency content input motion was represented by Loma Prieta event with critical frequency of 0.70:1.12Hz. Kobe records represented the intermediate A/V event with critical frequency of 1.45Hz while Northridge event with critical frequency of 4.64Hz was selected for high A/V ratio. Figure 3 depicts the time histories for the selected input motions. The structure response is investigated at the top of building at location A and at the surface of ground (location B) due to the excitation at bedrock (location C) as shown in Fig. 1.

Table 3 Input motion characteristics [26]

Event	Classification Criteria	Date	Maximum Acceleration	Maximum Velocity (m/s)	A/V	Critical Frequency (Hz)
Loma Prieta - California	Low A/V < 0.8	1989	0.1075g	0.198	0.543	0.70-1.12
Kobe -Japan	Intermediate 0.8< A/V<1.2	1995	0.836g	0.924	0.906	1.45
Northridge-California	High A/V > 1.2	1994	1.048g	0.754	1.389	4.64

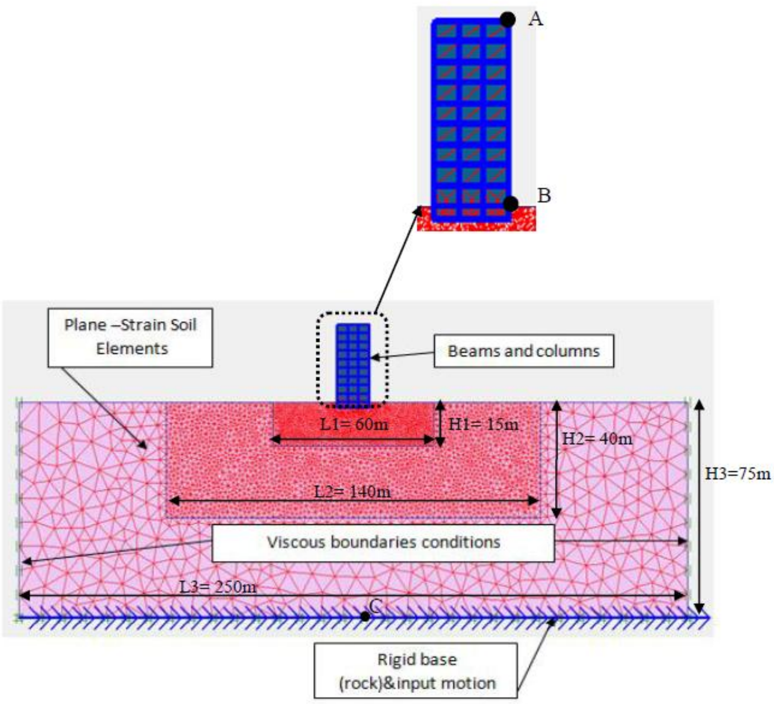


Figure 1 Soil-structure system and finite element modeling

Where:-

EA Axial rigidity of element (KN)

EI Flexural rigidity of element ($KN.m^2$)

d_{equ} Equivalent thickness of element

that is calculated from $\sqrt{(\frac{12EI}{EA})}$

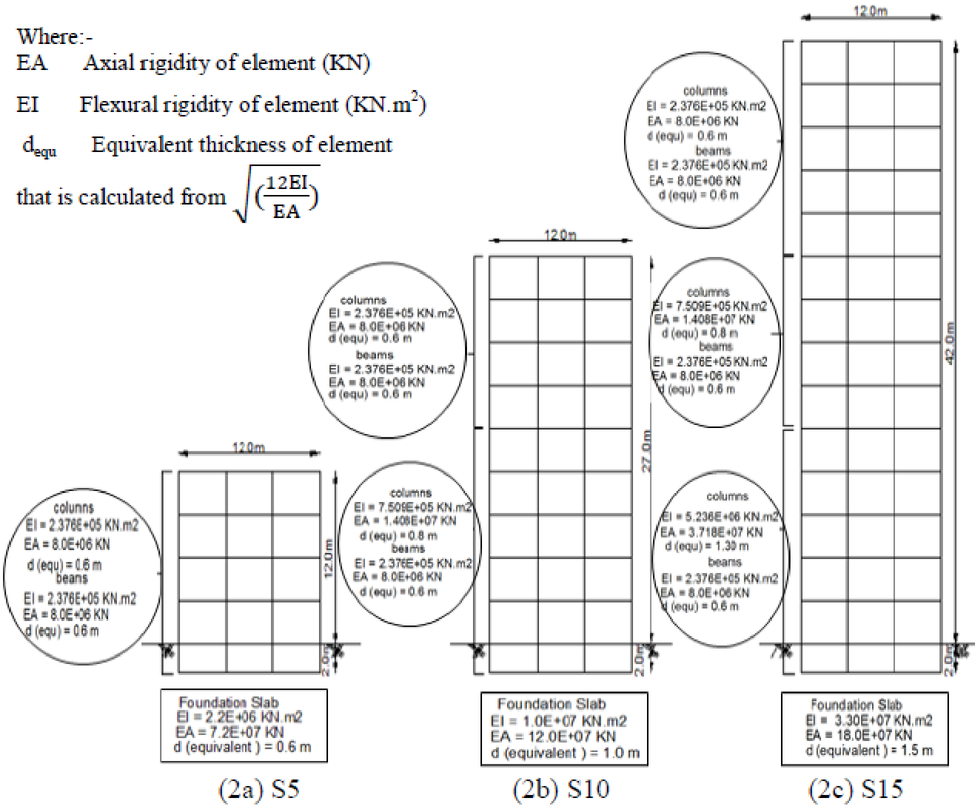


Figure 2 Dimensions and characteristics of the investigated frames,

(a) 5-story building S5, (b) 10-story S10 building and (c) 15-story building S15

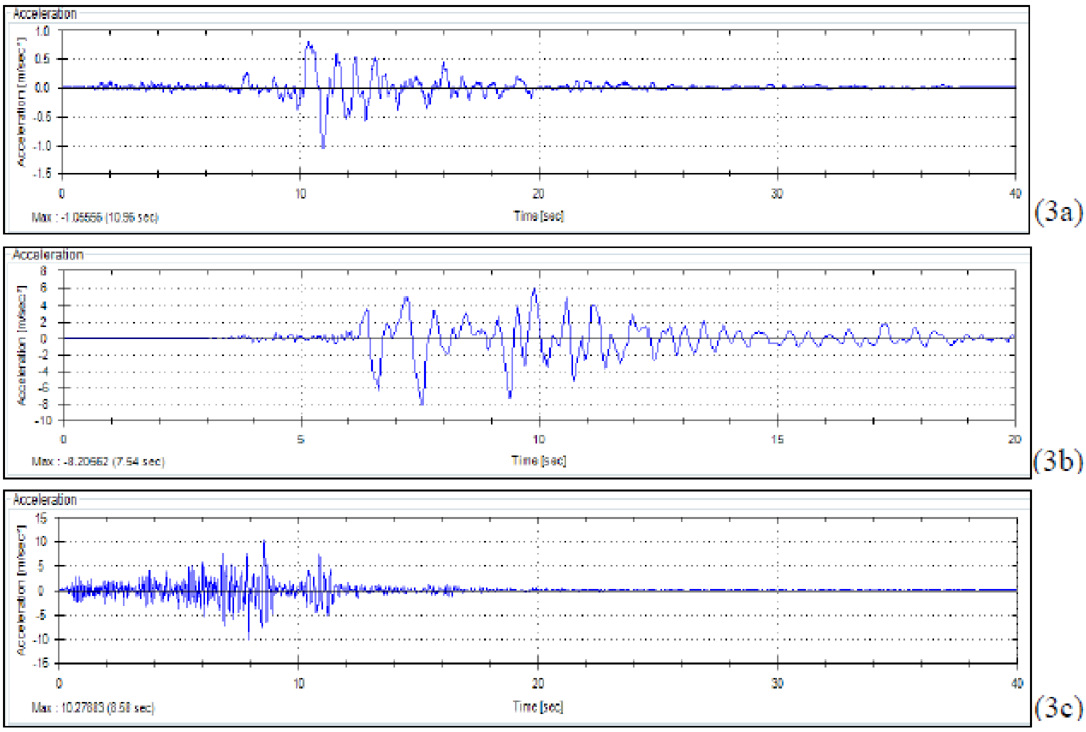


Figure 3 Acceleration time histories of the selected input motions, Loma Prieta 1989, (b) Kobe 1995 and (c) Northridge 1994

3. RESULTS AND DISCUSSIONS

3.1. Seismic Response at the Ground Surface

Since the acceleration is the most concerned response for the structure excitations, the maximum acceleration at location B is investigated in case of building exists and without building for loose soil condition (class D) to find out the effect of the building on the ground surface response. The resulted maximum accelerations at location B due to different input motions for S5, S10, and S15 story buildings are given in table 4. Concerning the case of without building, the maximum acceleration at B showed amplification ratio of about 200% compared with input motion of Loma Prieta earthquake that had low frequency contents and low acceleration amplitude of 0.1075g. For the case of Kobe input motion with intermediate frequency content and relatively high amplitude (0.836g), the acceleration at B resulted out with a reduced value as low as 73% of the input motion. Moreover, the resulted acceleration at B possessed very low value giving only 25% of the input motion of Northridge with high frequency contents and amplitude of 1.048g. These results indicated that the surface response is strongly dependent on the input motion characteristics and the soil conditions. During the excitations, the loose soil exhibited different strain levels leading to different levels of nonlinear hysteresis that resulted in different energy dissipation levels and damping ratios [4, 5, 9]. At high strain level due to Northridge excitation the energy consumption and damping reached a high level, consequently, the resulted surface acceleration amplitude is suppressed possessing the highest reduction. The opposite behavior occurred in case of Loma Prieta excitation where the low strain level with low frequency contents did not excite the soil nonlinearity leading to the amplified response at location B. Kobe input motion that is referenced as intermediate case showed actual intermediate behavior between the amplification ratio of 200% in case of Loma Prieta and the 75% reduction in amplitude in case of Northridge excitation. When the building exists, it is obvious that the resulted maximum acceleration possessed reduced values compared to the case of without buildings except for the case of S15 when subjected to Kobe excitation. Moreover, Kobe excitation resulted in slight reduction in the acceleration amplitudes in case of S5 and S10. These results reflect the mutual effects between the building, the underlying soil and the input motions showing that even the ground surface response can be affected by the building existence and the input motion characteristics.

Table 4 Maximum acceleration (m/s^2) at location B, soil class D

Building Ref. / Input motion	S5	S10	S15	without building
Loma Prieta	1.353	1.579	1.835	2.021
Kobe	5.513	5.833	6.381	5.960
Northridge	2.272	2.157	1.867	2.632

3.2. Variation of Structure’s Fundamental Frequency

The most important step for the seismic design of the structure is to determine its fundamental frequency in a simple way based on proper assumptions that can account for the affecting parameters. It is common for most of structural engineers to consider

the buildings as fixed at its base. This assumption is not proper since the underlying soil in the near field often consists of soft soil layers that possess different properties and may be drastically affect fundamental frequency of the soil-structure system. To simulate the effect of different boundary conditions on the fundamental frequency, the buildings under investigation have been analyzed assuming three different boundary conditions for the base: (i) fixed base, (ii) equivalent soil springs, and (iii) flexible base considering full SSI using Plaxis 2D. For comparison, the fundamental frequency (f_o) of each building is calculated using the simple equation given by the Egyptian Code ECP-201, 2012 [14] as:

$$f_o = 1/(Ct*H^{0.75}) \quad (1)$$

Where, Ct is a factor depends on the structural system (taken as 0.075 for moment resisting building frames) and H is total height of building. The equivalent soil static spring stiffness is calculated employing the Winkler spring approach as usually done by many authors [28-30] as follows:

$$K_z = \frac{2GL}{1-\nu}\left[0.73 + 1.54\left(\frac{B}{L}\right)^{0.75}\right] \quad (2)$$

$$K_y = \frac{2GL}{2-\nu}\left[2 + 2.5\left(\frac{B}{L}\right)^{0.85}\right] \quad (3)$$

$$K_x = K_y - \frac{0.2GL}{0.75-\nu}\left[1 - \left(\frac{B}{L}\right)\right] \quad (4)$$

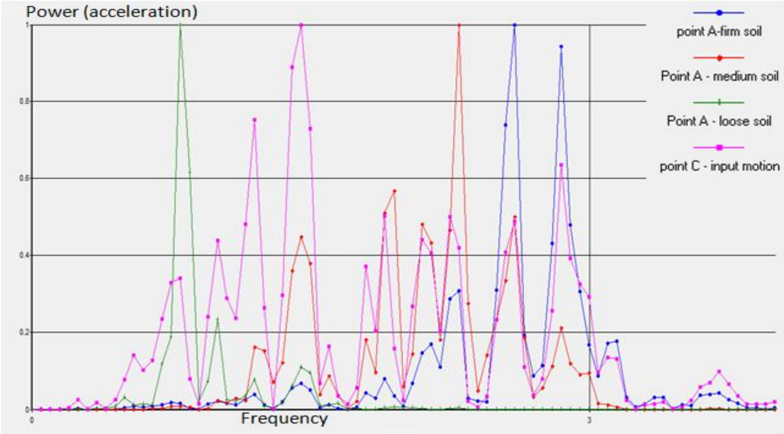
$$K_{ry} = \frac{G}{(1-\nu)x0.75}\left[3\left(\frac{L}{B}\right)^{0.15}\right] \quad (5)$$

Where, G & ν are the shear modulus and Poisson's ratio for the soil as given in table 1, respectively. L and B are the half of length and width of the foundation as given in table 2, respectively. K_z, K_y, K_x, and Kry are the vertical, horizontal in y-direction, horizontal in x-direction, and rocking around y-axis equivalent static stiffness of the soil springs, respectively. The calculated stiffness of soil springs for different soil conditions are summarized in table 5.

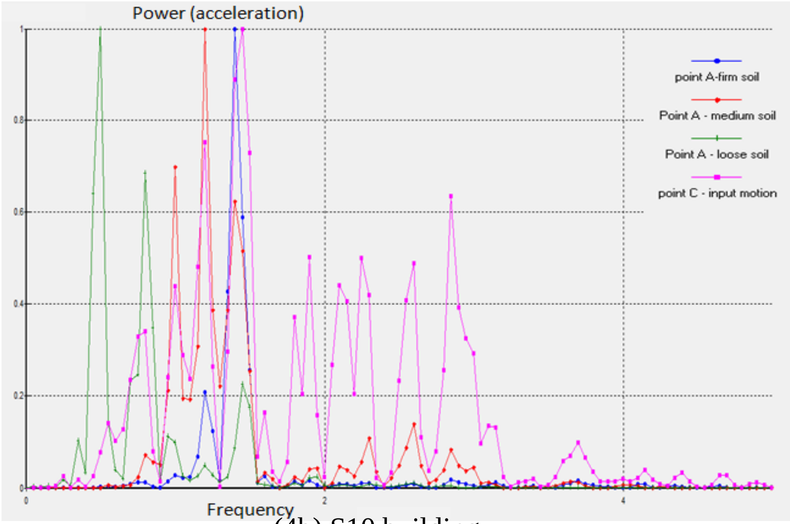
Table 5 Equivalent static spring stiffness for different soil classes

Soil condition	K_x (kN/m)	K_z (kN/m)	K_{ry} (kN/m^2)
Firm soil – class A	45.0 E+06	59.3 E+06	14.7 E+06
Medium soil – class C	28.8 E+05	36.2 E+05	9.0 E+05
Loose soil – class D	28.2 E+04	33.7 E+04	8.4 E+04

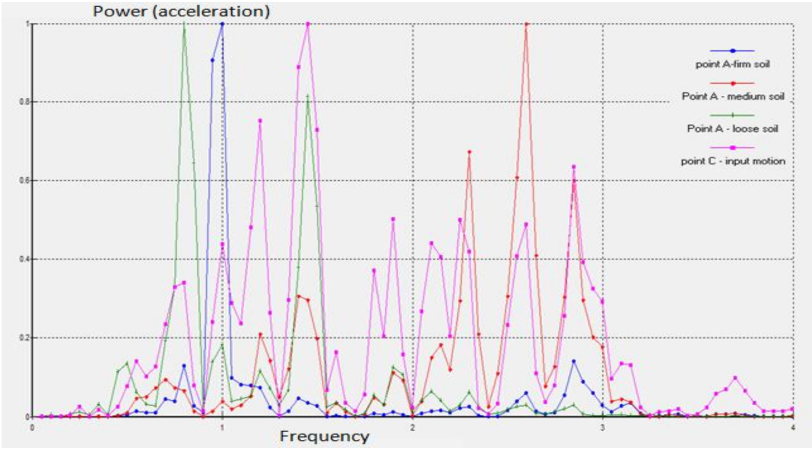
The very well-known structure analysis computer software SAP2000 V14 [31] is utilized to calculate the fundamental frequency of the investigated buildings assuming the structure is fixed at its base or resting on equivalent soil spring supports. In addition, the selected three input motions are applied at the bedrock with full SSI (using Plaxis 2D) assuming different soil conditions, then the Fourier power spectrum of the acceleration at top of the building (location A) is obtained for each case as shown in Fig. 4 for the case of Kobe input motion as a sample. Because the seismic response power spectrum is always populated over a wide range of frequency as it is affected by the input motion frequency contents, the critical frequency accompanied with the highest amplification of the power amplitude compared to the input motion is picked out and considered to be the fundamental frequency of the soil structure system with full SSI.



(4a) S5 building



(4b) S10 building



(4c) S15 building

Figure 4 Fourier power spectrum of acceleration at location A & C for different soil conditions due to Kobe input motion,

S5 building, (b) S10 building and (c) S15 building

The obtained fundamental frequencies for all boundary conditions along with those calculated by ECP-201 equation are summarized in table 6. The results showed

that the fundamental frequency obtained assuming fixed base condition using SAP2000 was in a very good agreement with that calculated by ECP-201(Eq. 1) for S5 building. The fundamental frequency of the fixed base S10 building was lower by about 14% while that of S15 building was lower by about 8% compared with ECP-201 regardless of the soil conditions. The fundamental frequency obtained considering equivalent soil springs showed good agreement with that of the ECP-201 with reduction of about 3% for S5 building resting on soil class A or class C. For S5 building resting on soil class D, the frequency obtained assuming equivalent soil springs was less than that of ECP-201 by about 13%. The frequency of S10 building was lower by about 15%, 17% and 25% for soil class A, class C and class D, respectively. Concerning S15 building, the fundamental frequency was reduced by about 9%, 11% and 23% when rested on soil springs of class A, class C and class D, respectively. As a general trend for the studied cases, the assumption of foundation flexibility through the equivalent soil static springs resulted in lower fundamental frequency (i.e. longer period) for all considered types of soil and building heights. The lower frequency (longer period) leads to lower spectral design acceleration, reduced seismic loads on the structure and, consequently, reduced base shear and element forces. The loose soil condition (class D) can reduce the fundamental frequency by about 13% for low rise building with five story and about 25% for mid-rise building with ten and fifteen stories.

Table 6 Fundamental frequencies for different boundary conditions

Building Reference	**Boundary Conditions**	**Fundamental Frequency f_o(Hz)**		
		Firm soil class A	**Medium Soil class C**	**Loose Soil class D**
S5	ECP-201, 2012	1.84		
	Fixed by SAP2000	1.81		
	Equivalent Springs	1.80	1.79	1.60
	Full SSI (Plaxis 2D)	2.59	2.05	0.79
S10	ECP-201, 2012	1.06		
	Fixed by SAP 2000	0.917		
	Equivalent Springs	0.90	0.88	0.79
	Full SSI (Plaxis 2D)	1.32	1.02	0.42
S15	ECP-201, 2012	0.78		
	Fixed by SAP 2000	0.72		
	Equivalent Springs	0.71	0.70	0.60
	Full SSI (Plaxis 2D)	0.95	0.65	0.80

The fundamental frequencies obtained by more realistic full SSI models using Plaxis 2D were higher than that of the Egyptian code (Eq. 1) for the buildings when rested on firm soil (class A). The frequency was increased by about 40%, 24% and 21% for S5, S10 and S15 building, respectively. For relatively low-rise S5 buildings, the natural period was very small (less than 0.4 second) that may lie very close to the highest spectral acceleration ordinate. Also, for S10 and S15 building on firm soil, the

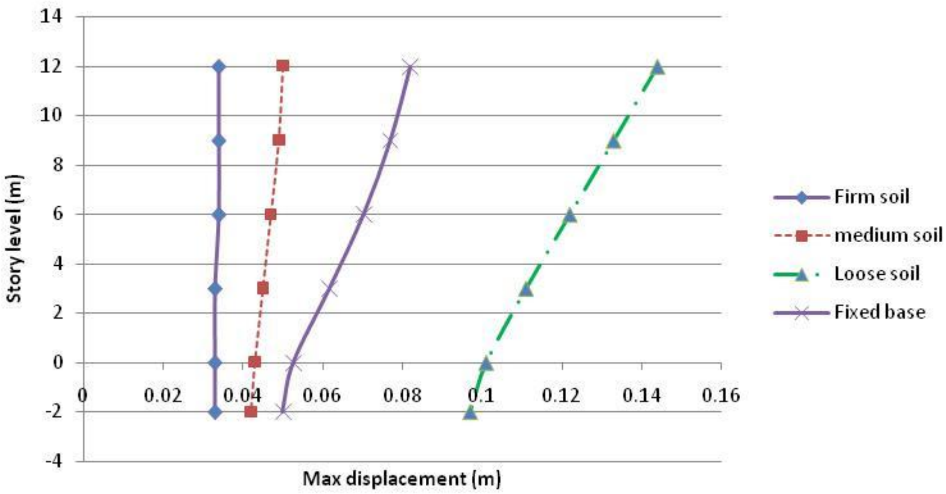
higher frequency (shorter period) will lead to higher spectral acceleration. When the intermediate soil (class C) is considered, the fundamental frequency was increased by about 11% for S5 while reduced by 4% and 17% for S10 and S15 building, respectively. However, the loose soil condition (class D) drastically reduced the fundamental frequency by 57% and 60% for S5 and S10, respectively, reflecting the effect of soil nonlinear behavior. However, the Egyptian Code ECP-201 allows for 20% only as a maximum increase in natural period (i.e. about 17% reduction of the fundamental frequency) when calculated by other approaches rather than equation (1) [14]. Therefore, based on the obtained results of studied cases, it can be concluded that the effect of soil-structure interaction is more significant for low rise building (S5) when rested on firm soil (class A) and for mid-rise buildings S10 and S15 when rested on medium and loose soil conditions (classes C & D). Moreover, employing the equivalent static springs could not precisely represent the underlying soil when the soil-structure system subjected to seismic excitations.

3.3. Seismic Response of Structures

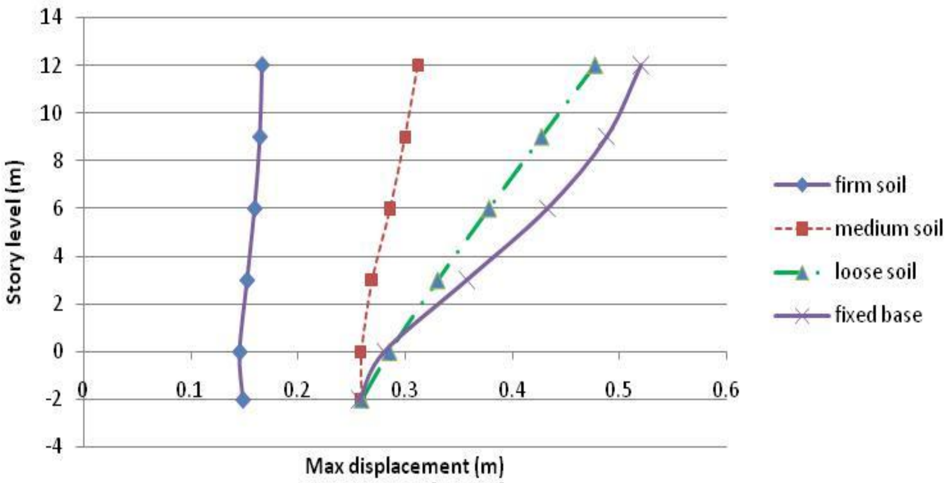
The acceleration amplitude at the top of each building (location A) is recorded for each case of input motion and soil class. The ratio between the resulted amplitude and the corresponding amplitude at the bedrock (location C) is then calculated and summarized in table 7. In case of soil class C, the amplitude ratio possessed a value of 3 of more for S5 and S10 building due to Loma Prieta input motion and S5 building due to Kobe motion. In case of soil class A, the ratio exceeded the value of 2 for S5 building due to Northridge and for S15 building due to Loma Prieta excitation. The amplitude ratio fall below the unity in case of Northridge input motion and soil class C possessing a value of 0.618 for S10 and 0.749 for S15. The very low values of the amplitude ratio resulted in case of loose soil class D where a value of 0.3 resulted for S5, 0.179 for S10 and 0.208 for S15 building. Also low amplitude ratios of 0.606 due to Kobe excitation resulted for S10 and 0.571 for S15 on soil class D. The above results confirm our previous finding that the high strain level caused by Northridge excitation lead to the high energy dissipation and damping by the soil nonlinearity resulting in suppressed acceleration in case of medium to loose soil conditions. Also, the almost linear behavior of soil at low strain level in case of Loma Prieta motion that had low amplitude and frequency contents is confirmed.

Table 7 Acceleration amplitude ratio at top of buildings

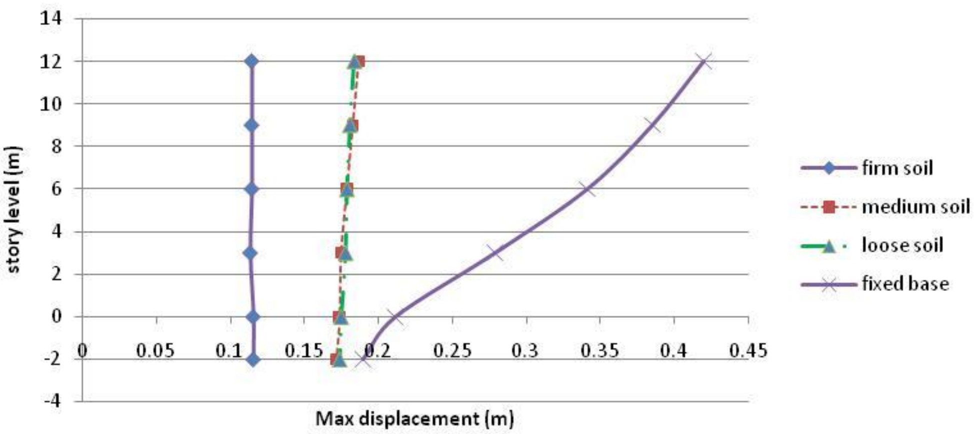
Building Reference	Input motion	Firm soil class A	Medium soil class C	Loose soil class D
S5	Loma Prieta	1.270	3.060	2.970
	Kobe	1.760	3.010	1.090
	Northridge	2.145	1.190	0.300
S10	Loma Prieta	1.677	3.193	1.501
	Kobe	1.645	1.622	0.606
	Northridge	1.008	0.618	0.179
S15	Loma Prieta	2.199	1.852	1.280
	Kobe	1.322	2.465	0.571
	Northridge	1.171	0.749	0.208



(5a) Loma Prieta



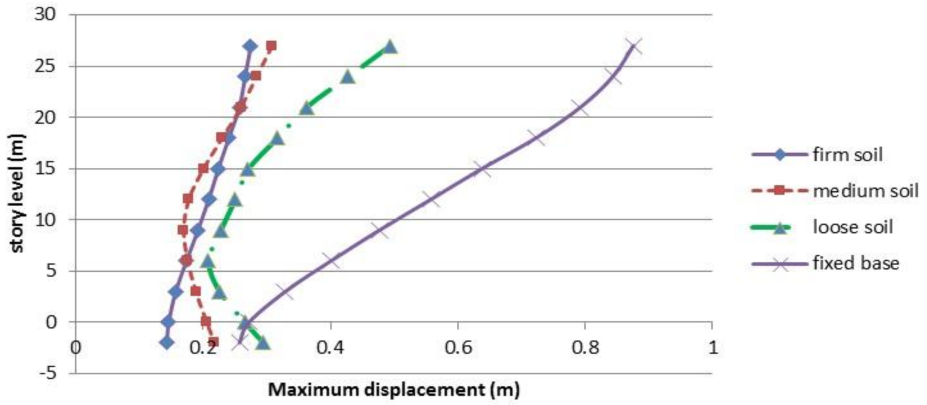
(5b) Kobe



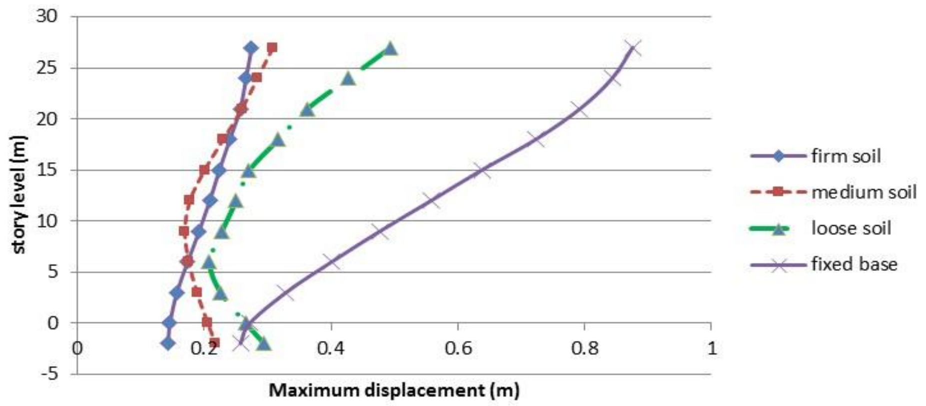
(5c) Northridge

Figure 5 Lateral displacement of S5 building considering full SSI and fixed base due to different input motions,

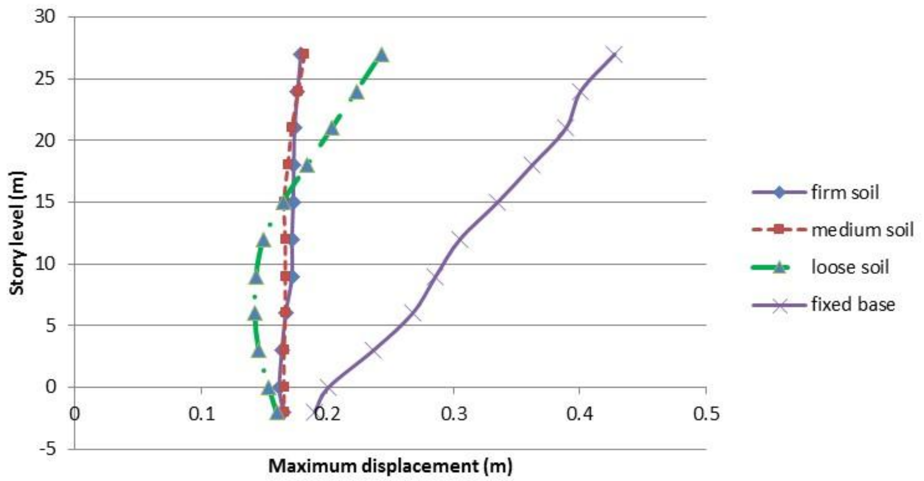
Loma Prieta, (b) Kobe and (c) Northridge



(6a) Loma Prieta



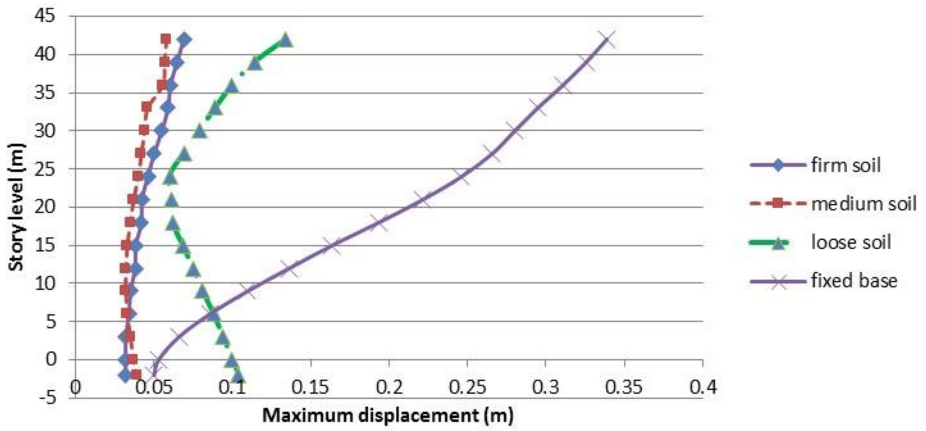
(6b) Kobe



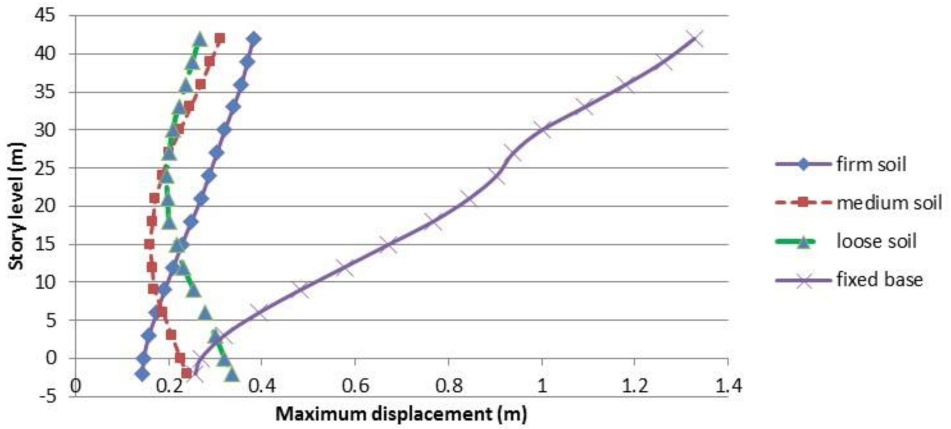
(6c) Northridge

Figure 6 Lateral displacement of S10 building considering full SSI and fixed base due to different input motions,

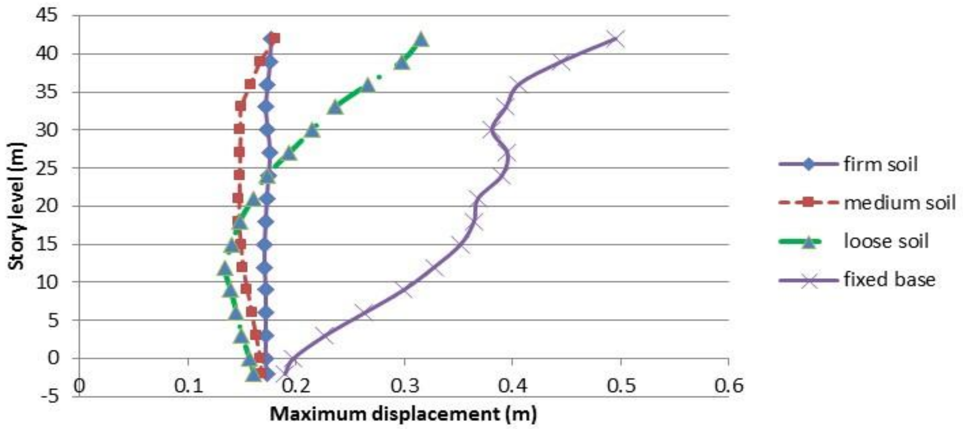
Loma Prieta, (b) Kobe and (c) Northridge



(7a) Loma Prieta



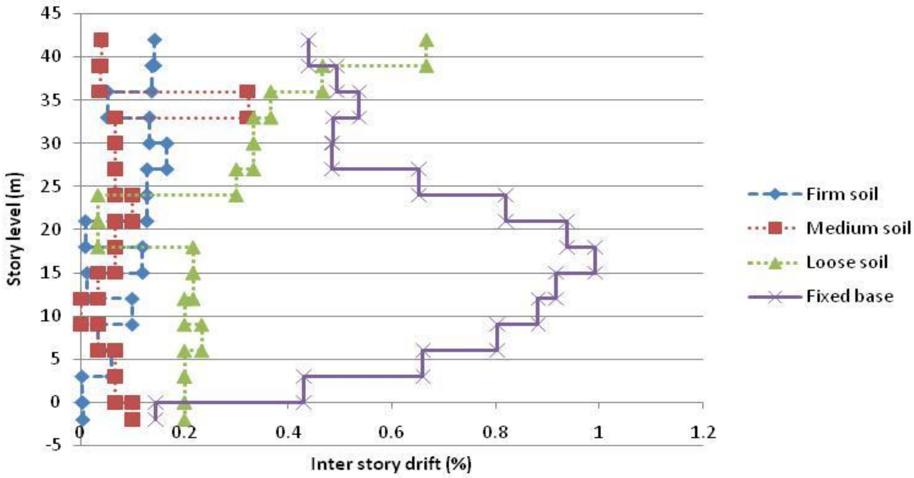
(7b) Kobe



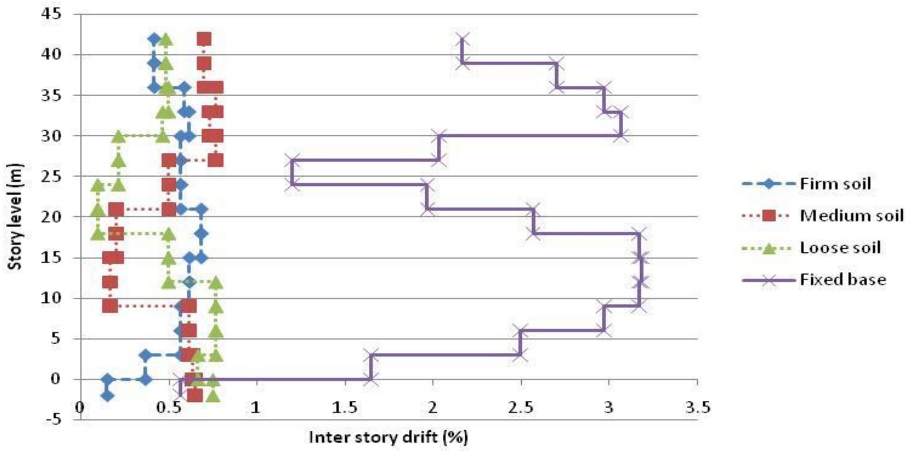
(7c) Northridge

Figure 7 Lateral displacement of S15 building considering full SSI and fixed base due to different input motions,

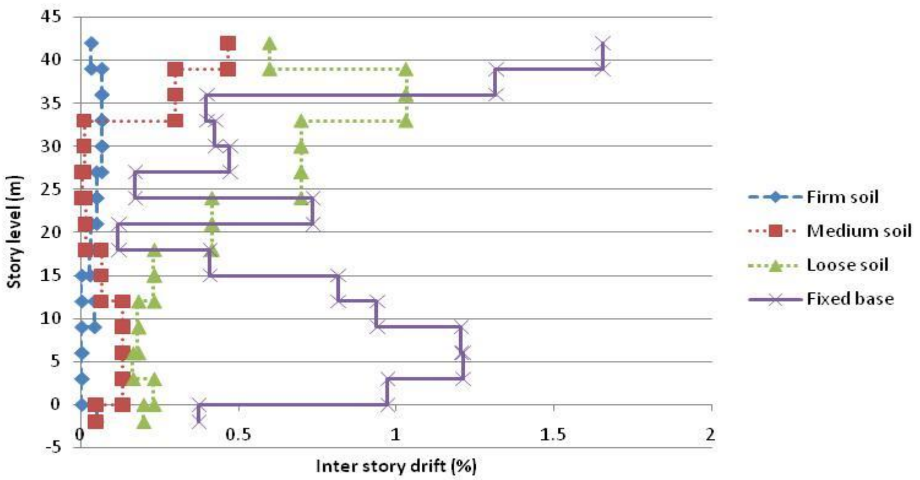
Loma Prieta, (b) Kobe and (c) Northridge



(8a) Loma Prieta



(8b) Kobe



(8c) Northridge

Figure 8 Inter-story drifts of S15 building considering full SSI and fixed base due to different input motions,

(a) Loma Prieta, (b) Kobe and (c) Northridge

The total lateral displacements along the height of outer column of each building are given in Figs. 5 through 7 due to different input motions when considering full SSI along with the case of fixed base assumption. It can be noted that the displacement at the foundation level (-2.0m) possessed the same value for all studied building types (S5, S10, S15) when subjected to the same input motion assuming fixed base condition. However, when full SSI condition is considered, the resulted displacements had different values at the foundation level even for the same building depending on the class of underlying soil. Almost, in all studied cases, the firm soil condition (class A) resulted in the lowest values of the foundation level displacements. On the other side, the loose soil condition (class D) resulted in the highest displacement values at foundation level that even exceeded the resulted values in case of the fixed base assumption except for the case of Northridge excitation. Hence, changing the building height and/or the input excitation resulted in different values of displacements at foundation level reflecting the effects of the mutual interaction between the three considered parameters, the soil, the building and the input motion.

When the lateral displacement (drift) at the top of each building is considered, the assumption of fixed base condition resulted in the highest displacement in all studied cases except for the case of S5 on soil class D excited by Loma Prieta motion. However, the assumption of full SSI drastically reduced the lateral drift at the building top, especially, in the cases of moderate and high frequency contents (A/V) input motions of Kobe and Northridge, respectively, where the nonlinear soil behavior was highly expected. The drift reduction was more pronounced for all cases of S10&S15 buildings and case of S5 building under Northridge excitation. As a general behavior for all studied cases, the loose soil condition (class D) resulted in higher displacement than the case of soil class A and class C except for the case of S15 when subjected to Kobe motion. The displacement distribution along the building height revealed that almost the simple cantilever mode was excited for all buildings when considered with fixed bases or supported by firm soil (class A). However, the cases of buildings resting on soil class C and class D resulted in different distribution indicating that higher modes of vibrations were also excited. In the seismic design of buildings, the serviceability is an important issue that should be assured and many codes and provisions require certain restrictions not only on the total drift but also on the inter-story drift that directly leads the story shear force. Figure 8 depicts the resulted inter-story drift for S15 building as a percentage of the story height when excited by the considered three input motions. As can be easily noted, the assumption of fixed base condition resulted in the highest inter-story drifts. The loose soil conditions (class D) resulted in higher drift percentage than the other two soil classes.

For the structural designer, the total seismic base shear to be resisted by the columns and transmitted to the foundation is the most concerned and governing action. The resulted total base shear for each building with fixed base is summarized in table 8 as a reference value for each study case. The resulted base shear with consideration of SSI is given as a ratio of the corresponding fixed base reference value. The assumption of SSI resulted in reduced value of the base shear in all cases except for S5 and S15 when supported by soil class A and excited by Loma Prieta motion. The firm soil (class A) condition resulted in reduction in the base shear varied between 20% to 68% of the fixed base condition depending on the input motion and the building type. The reduction varied between 11% and 82% in case of medium soil condition (class C). The case of the loose soil condition (class D) resulted in reduction varied from 32% to 93% of the fixed base reference value. For S5 building,

the lowest reduction in base shear was 11% in case of medium soil condition and Loma Prieta input. The highest reduction was about 92 % in case of loose soil and Northridge input. For S10 building, the lowest reduction was 23% in case of firm soil under Loma Prieta input while the highest reduction was 89% under Northridge input with loose soil condition. The lowest reduction for S15 building 21% for firm soil and Northridge input motion. The overall highest reduction was 93% for S15 building when supported by soil class D and excited by Northridge. These results clearly indicate that it is crucial to consider the SSI effects in the seismic design of mid-rise buildings as it may lead to great reduction of the governing design forces without any aggression against the structure safety.

Table 8 Variation of total base shear forces due to SSI effects

Building Reference	**Input motion**	**Fixed base shear force (V_f)kN**	**Firm soil ratio $\tilde{V}_A/V_f$**	**Medium soil ratio $\tilde{V}_C/V_f$**	**Loose soil ratio $\tilde{V}_D/V_f$**
S5	Loma Prieta	750.34	2.148	0.889	0.681
	Kobe	6391.42	0.318	0.514	0.227
	Northridge	6503.36	0.453	0.228	0.076
S10	Loma Prieta	2249.72	0.769	0.470	0.248
	Kobe	6594.12	0.616	0.581	0.230
	Northridge	5454.12	0.630	0.298	0.107
S15	Loma Prieta	3045.48	1.088	0.314	0.190
	Kobe	12507.66	0.724	0.381	0.155
	Northridge	12078.74	0.792	0.180	0.071

4. CONCLUSIONS

Numerical investigation of seismic response of mid-rise buildings with five, ten and fifteen story with consideration of full soil structure interaction effect is presented. Three different soil conditions and three different input motions were considered for each building. The analysis was performed utilizing Plaxis 2D software. The obtained results were compared with the results obtained when the buildings were assumed fixed at their base or the resting on equivalent static soil spring. Based on the obtained results, the following conclusions can be stated:

1. The building existence reduced the ground surface acceleration amplitude to different extents depending on the building height, soil type and the input motion characteristics.
2. The high frequency contents and large amplitude of the input motion could excite the soil nonlinearity leading to high energy dissipation and damping ratio, and consequently, substantial suppression of the surface acceleration. The resulted reduced acceleration could be as low as 25% of the input motion in case of soil class D.
3. The fundamental frequency of the five story building with fixed base structure was in a good agreement with the Egyptian code simple formula result. For ten and fifteen story buildings, the fixed base fundamental frequencies was lower than the Egyptian code by 14% and 8%, respectively.
4. As a general trend for the studied cases, the assumption of foundation flexibility through the equivalent soil static springs resulted in lower fundamental frequency (i.e.

longer period) for all considered types of soil and building heights. The loose soil condition (class D) can reduce the fundamental frequency by about 13% for with five story and about 25% for building with ten and fifteen story.

5. The assumption of full SSI resulted in higher fundamental frequency for buildings rested on firm soil (class A). The ratios of increase were about 40%, 24% and 21% for S5, S10 and S15 building, respectively. However, the loose soil condition (class D) drastically reduced the fundamental frequency by 57% and 60% for S5 and S10, respectively. Therefore, employing the equivalent static springs could not precisely represent the underlying soil when the soil-structure system subjected to seismic excitations.
6. The nonlinearity of medium and loose soil conditions (class C&D) resulted in highly reduced response at the building top compared with the fixed base assumption when subjected to intermediate (A/V) and high (A/V) excitations. The loose soil conditions (class D) resulted in higher drift percentage than the other two soil classes. However, the assumption of fixed base condition resulted in the overall highest inter-story drifts.
7. The loose soil condition (class D) resulted in reduction in the total base shear varied from 32% to 93% of the fixed base reference value. The reduction varied between 11% and 82% in case of medium soil condition (class C). The firm soil (class A) condition resulted in reduction varied between 20% to 68% of the fixed base condition depending on the input motion and the building type.
8. The results clearly indicated that it is essential to consider full SSI effects in the procedures of the seismic design of mid-rise buildings as it may lead to great reduction of the governing design forces without any aggression against the structure safety.

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